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EARTHQUAKE BEHAVIOUR OF MODERN TIMBER CONSTRUCTION SYSTEMS

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ABSTRACT: Several modern timber construction systems outside the timber frame system have become accepted throughout Europe in the past few years. Comprehensive shear wall testing of two modern timber construction systems was carried out at KIT. Additional tests on timber frame construction provided the possibility to compare the test results of the modern systems and the "traditional" timber frame construction. Numerical modelling was performed on both systems to study the real-time domain performance and finally investigate the basics for force-based design.

KEYWORDS: Earthquake, Seismic, Hysteresis, Behavior Factor q, Timber frame

1 INTRODUCTION

Based also on their environmental sustainability, timber constructions represent a growing number of today's residential houses throughout the world. The use of renewable resources, the comfortable climate inside timber buildings and a favourable cost-performance ratio led to a general acceptance of these buildings.

Most of timber residential houses are timber frame constructions, which are widely investigated in almost any aspects of construction. In the past few years several modern timber construction systems outside the timber frame system were developed throughout Europe.

Manufacturing progress brought a general possibility of using novel construction materials like X-lam. Innovations regarding building physics or simple assembling and finishing of the building led to other innovative timber constructions, two of them are presented in this paper.

Shear wall tests on these systems were carried out by Karlsruhe Institute of Technology (KIT). Also some tests on timber frame construction were carried out in order to compare the outcomes of the modern systems and the "traditional" timber frame construction. Due to the large amount of mechanical fasteners used in timber frame constructions, their behaviour in repeated loading conditions is very ductile and hence favourable for earthquake loading. The investigated modern timber construction systems also contain a multitude of mechanical fasteners and some other promising characteristics for earthquake behaviour are given as well. For these reasons the systems presented in this paper are very suited for seismic active areas. A couple of basics for the common use in earthquake prone regions were developed through the recent research work.

Finally, numerical modelling was performed on the two systems to determine the behaviour factor, which is the basis for force-based design.

2 NOVEL TIMBER CONSTRUCTION SYSTEMS

Some general information about the studied systems is given in the following. Both manufacturers are originated in southern Germany and both systems have a general technical approval which currently does not include earthquake design.

2.1 X-LAM MASSIVE PANEL SYSTEM

The X-lam massive panel system which was invented by Lignotrend Company (www.Lignotrend.de) is made up of cross-laminated timber panels of 0.625 m x 2.5 m x 0.09 m (length x height x thickness). The panels consist of load-bearing sawn timber members in a 125 mm grid. Crosswise lamination results in a closed load bearing layer on the outer panel side, an open grid on the inner side can be used for installation. Within certain limits the arrangement of the timber boards is variable, so that the installation channels can be arranged to the desired location already in the factory (Figure 1 a) and b)). To produce an entire wall, the panels are mounted on associated top and bottom rails already in the factory.

The two outermost vertical boards of the closed layer are omitted. The resulting space at the vertical panel edges are then fitted with boards connected to both panels with mechanical fasteners that transfer shear forces between elements when the wall is subjected to horizontal loads.

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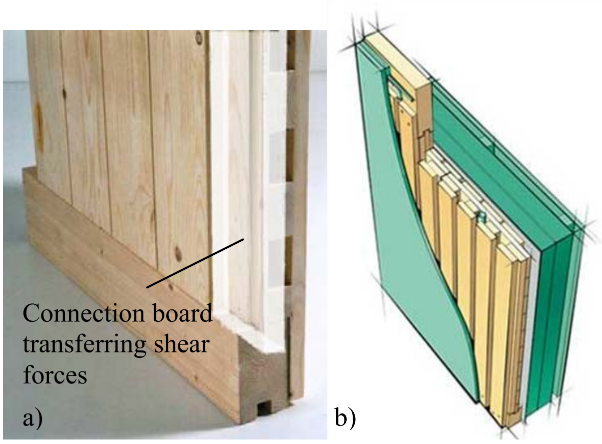

__Figure 1:__ a) Close-up view of the bottom of a single X-lam massive panel, b) Panel with inner finish plus installation and outer insulation

The shear boards are joined to the panels by mechanical fasteners such as staples or grooved coil-nails. The boards are made of solid timber or plywood.

__Figure 2:__ Building made of X-lam massive panels

Openings for doors and windows are spared already in the factory. The accomplishment is done by placing an associated top rail on the surface of the wall. The vertical timber boards of the panels overlap on both the top and the bottom side. These overlaps are also connected to the bottom and top rail with the same mechanical fasteners that are used for the shear boards. Whole walls, corresponding floor systems, and consequently whole buildings can be pre-assembled to a great extent. Wall sizes are primarily limited by transportation requirements, the erection of buildings thus is very fast (Figure 2). The building shell usually is set-up in 1-2 days. Predrilled installation channels contribute to low costs and high flexibility for the completion of the interior.

2.2 PREFABRICATED TIMBER WALL ELEMENTS

The main feature of Prefabricated Timber Wall Elements (PFTE) is prefabricating wooden "brick" elements in small units primarily using sawmill residues. These elements represent a simple, sustainable construction system which is easy to handle on the building site (www.hib-system.com). The mass of a single element is less than 25 kg, it can be moved by hand and walls can be built without a crane. The "wooden brick" in its basic dimension of 1,0 m x 0,5 m x 0,16 m (length x height x thickness) consists of four solid wood columns and chipboard sheathing (Figure 3).

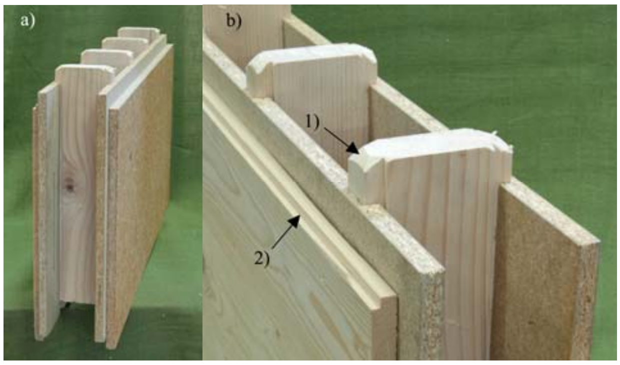

__Figure 3:__ a) Prefabricated timber wall element b) Offset of columns (1) offset of layers (2)

The wood columns are connected by dove tails to the (inner) chipboard layers. On the one hand this means a simple and close connection between the inner sheathing layer and the columns; on the other hand it allows the columns of the lower element to slide into the sheathing of the element on top. The single elements are stuck together by these overlapping/ shortened columns with dove tail geometry at the top/ bottom of the element. The overlapping/ shortening of the columns gives the wall initial stability. On both sides a second layer is fixed to the inner sheathing layer.

__Figure 4:__ Building made of prefabricated timber wall elements

The second (outer) layer consists of chipboard on the subsequent inner side of the building and of timber boards on the subsequent outer side of the building. The second sheathing layer is fixed with a horizontal and vertical offset of 30 mm. When setting up the wall the offset of the outer layers of lower and upper elements slide into the next one, so that the outer layer overlaps from one element to another. After finishing erection the

overlapping parts of the sheathing are connected on the inner side of the building by pneumatically driven-in staples to create continuous shear walls (Figure 4).
The elements are available in wall thicknesses of b = 160 mm, b = 240 mm or b = 300 mm. The columns are spaced 250 mm; the hollow space between the columns can be used for insulation or installation. When erecting a wall with PFTE, first a wall plate is fixed to the foundation. The next layers are simply laid by stacking the wooden "bricks". When the planned wall height is reached, a continuous vertical stud is inserted from the top at least every 3 m of wall length. The vertical studs transfer the in-plane uplift forces to the foundation and they provide bending stiffness for loads perpendicular to the wall plane, e.g. wind loads. At the top of the wall the top rail is put into position and the vertical studs as well as the top rail are connected to the elements via self-tapping screws.

2.3 OTHER NOVEL SYSTEMS AND CONVENTIONAL TIMBER FRAME SYSTEM

In addition to the previously mentioned construction systems, other novel systems were recently developed. Probably best-known is the X-lam system, which in an excellent manner uses the advantages of timber construction: Due to cross-wise lamination swelling and shrinking is nearly prevented. The amount of massive timber leads to excellent load-carrying capacities and good climate properties. X-lam buildings for the most part can be customized in the factory; hence erection of these buildings is also fast. The connection of the X-lam panels is mostly carried out with self-tapping screws. When building in seismic active regions, dissipative zones can be designed by cutting the X-lam and to reconnect the elements using mechanical fasteners.
Several advantages are offered by conventional timber frame constructions. Flexibility in construction is also given as well as good building physics and sustainability. When subjected to seismic loads, the behaviour of timber frame construction is favourable. Shake table tests on a six storey building recently have shown the excellent behaviour of wood-frame construction when subjected to earthquake loads.

3 SHEAR WALL TESTS

3.1 TEST SETUP

The build-up of new test equipment for shear wall tests was part of the research project. In the past years, the topic of the application of realistic boundary conditions in shear wall tests was repeatedly discussed [1]. An existing testing machine with two hydraulic jacks for applying vertical loads was incorporated into the new wall testing facility. The hydraulic jacks for the vertical loads are either force or displacement controlled, so that different boundary conditions can be applied.
The centre of the wall top plate is attached to the horizontal hydraulic jack. Attaching the centre of the top plate allows the wall to freely rotate while always keeping the hydraulic jack nearly horizontal.
All tests were carried out using a force-controlled vertical load. This means that the top rail can rotate and slightly move, this is the boundary condition assumed to appear in most small and medium-rise timber buildings. The lightweight structures of such buildings, allows rotation of the walls.

3.2 TEST PROCEDURE AND STANDARD

Until to date no uniform testing procedure for both monotonic and cyclic testing of wooden shear walls is established. The existing procedures either govern monotonic or cyclic testing of either complete walls or small specimens. Several load protocols for the cyclic loading of walls and specimens do exist and there is no agreement about the application of realistic boundary conditions.
Probably best-known for the monotonic testing of wooden shear walls is EN 594 [2] which deals with the racking strength and stiffness of timber frame wall panels. Two different standards including cyclic load protocols which are used for testing mechanical fasteners are denoted in EN 12512 [3] and ISO 16670 [4]. Additional load protocols throughout the world do exist. Both, [3] and [4] are originally intended to carry out tests on connections with mechanical fasteners; no standard covers the testing of whole wall specimen.
All tests described in this paper were performed according ISO/CD 21581 [5]. This ISO committee draft was introduced in 2007 to provide a test method which is appropriate for classification and evaluation of shear walls in timber buildings. The static test according to EN 594 is included in ISO/CD 21581 as well as the cyclic displacement schedule described in ISO 16670. Depending on whether mainly the shear response of the wall or mainly the rocking (rigid body rotation of the wall) response of the wall should be reached, two methods of applying the boundary conditions are given. The reversed cycles in [5] are applied in terms of percentage of the wall's ultimate displacement determined from the static test according to [5] as well. Using the ultimate displacement, no yield displacement (which is difficult to determine because of different definitions) is needed. The ultimate displacement is defined a) as the displacement at failure or b) the displacement at 80 % of F_{max} in the descending part of the load-displacement curve or c) the displacement reaching H/15 (where H is the wall height), whichever occurs first.
At the moment, no approach to determine the energy dissipation of the wall is given in [5]. However it is mentioned that in future there may be a need to determine such additional properties. To gain some information about the energy dissipation of the tested walls, the equivalent hysteretic damping v_{ed} according to EN 12512 [3] is used. Values can be found in Table 1.
Some tests were performed using different vertical loads (1 kN/m, 10 kN/m or 20 kN/m) while the vertical load of 10 kN/m is regarded as a common vertical load for testing. Using higher vertical loads would lead to higher horizontal load-carrying capacities and would change the behaviour of the specimens to a shear failure. Using the low vertical load of 1 kN/m, the horizontal load-carrying capacities decrease distinctly and the hysteresis shape is extremely pinched, so that the amount of energy dissipation drops as well. The whole test thus becomes too

conservative. A vertical load of 10 kN/m is considered to be the appropriate value to achieve realistic test results.

3.3 TESTS ON X-LAM MASSIVE PANEL SYSTEM

The numerous mechanical fasteners in combination with the stiffness of the X-lam panels promised favourable results when subjected to static and cyclic loading. Different types of mechanical fastener are possible to attach the shear boards to the panels; however, the drive-in speed is the most important economic criterion. Therefore usually staples are used to connect the panels.

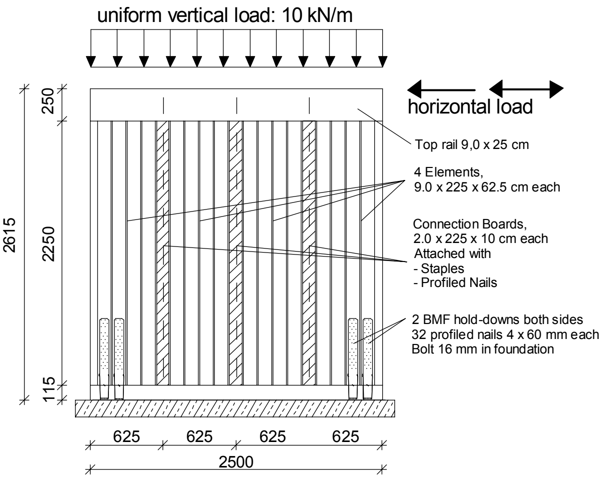

***Figure 5:** X-lam massive panel Test Wall Specimen*

Since staples are not generally used throughout Europe, also tests with nails were carried out. While the static load-carrying capacity of an X-lam wall specimen is only slightly influenced by the fasteners used, the behaviour under cyclic loading is strongly affected by the fastener type. While staples showed pronounced ductile behaviour in all tests, nails tended to break apart at multiple repeated cycles. Solid timber boards can be used when only minor horizontal loads, thus low shear forces have to be transferred by the connection boards. To avoid splitting caused by the alignment of fasteners, plywood boards were used. During the tests the use of 1.83 x 64 mm staples showed to be most efficient for the quick mounting of the panels, 2.8 x 65 mm grooved nails were used as an alternative. Test specimens are shown in Figure 5 and Figure 6.
The first tests were carried out with hold-downs conventionally used in Germany. The load-carrying capacity of the conventional hold-downs was, however, insufficient for the system at large displacements. Hence in the following tests, stronger hold-downs, which are commonly used in seismic prone areas, were used. Due to the high load-carrying capacity of the wall, two hold-downs on either side of the wall had to be employed. Due to the fact that realistic boundary conditions are postulated in ISO/CD 21581, commercially available hold-downs, which were connected to the panels by ringed-shank nails, were used.
The monotonic tests showed a pronounced ductile behaviour paired with an adequate stiffness. The maximum horizontal load (Tests at 270, 280 and 290 mm in Figure 8) is higher than the values reached with PFTE and timber-frame construction while the stiffness values for the proper ground connection are about to be the same.

***Figure 6:** a) X-lam massive panel Test Wall Specimen b) failure due cracking of boards when using nails c) deformed wall (top rail perspective)*

When subjected to cyclic loads, both, nail and staples led to adequate stiffness and high load-carrying capacities. When using staples, the displacement at maximum load in all cases reached values of 65 mm and more which corresponds to more than 2.5 % of the storey height in the tests. This displacement was also reached in most of the tests with nails while the corresponding maximum load for both configurations exceeds 30 kN/m.
The measured hysteresis equivalent viscous damping ranges from 10-15% (Table 1 and Figure 11).

3.4 TESTS ON PREFABRICATED TIMBER WALL ELEMENTS

In a research project at KIT [6] the PFTE building system was tested to determine its shear wall capacities with regard to its suitability for seismic regions.
Since there is no continuous sheathing of the wall being composed of several smaller elements, the main attention was focussed on the connections between the single elements.

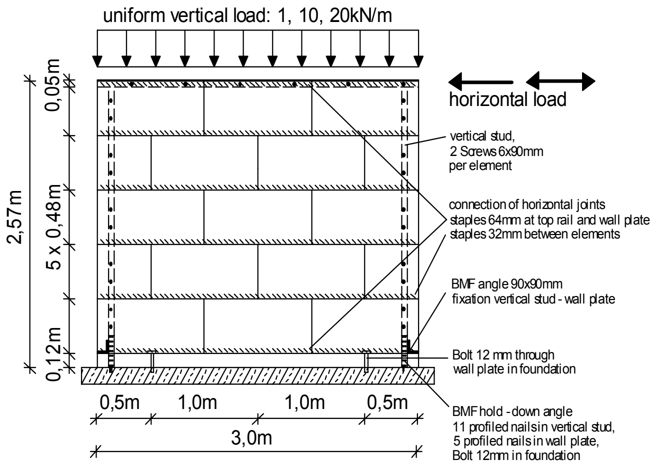

***Figure 7:** PFTE Test Wall Specimen*

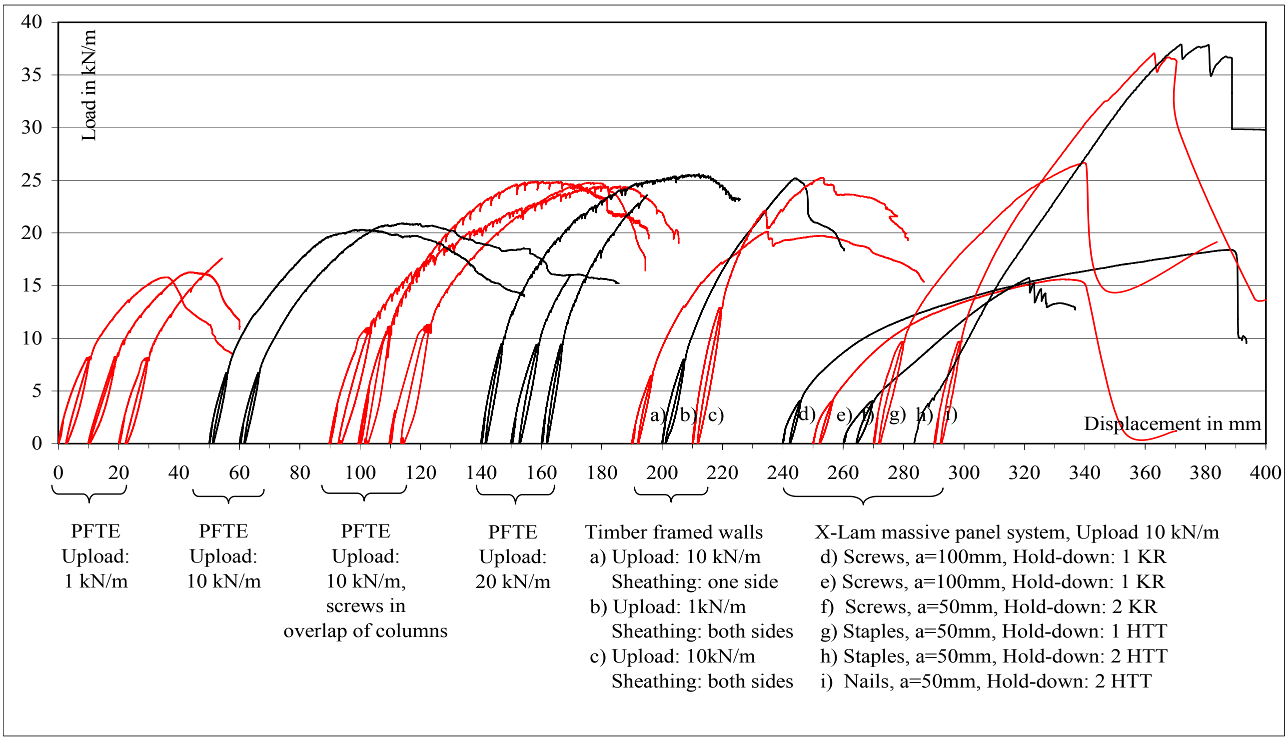

Figure 8: *Comparison of the monotonic tests of the different systems*

All the connections described in Figure 7 are of prime importance for the behaviour of the wall under different loading conditions. The staples used for the connections are very slender fasteners which bend easily. The plastic behaviour of the staples in bending as well as of the timber under embedding stresses lead to a ductile behaviour of the connections.

The friction between the elements causes additional energy dissipation, so a favourable behaviour under cyclic loading was promised.

The test setup for the PFTE shear wall tests is shown in Figure 7. All tests were performed using a PFTE wall thickness of 160 mm. In total, 11 monotonic tests with different uploads were performed with PFTE. The tests showed a pronounced ductile behaviour under static loads; however the failure mechanism of the whole wall specimen is strongly influenced by the upload (Figure 8). When applying vertical loads of 1 kN/m and 10 kN/m, tensile failure of the joints (Figure 9 a) and b)) are observed. When using an upload of 20 kN/m, the boundary condition changes to a shear wall behaviour which leads to a shear failure caused by sliding of the first joint (Figure 9 c)).

3.5 TESTS ON CONVENTIONAL TIMBER FRAME WALLS

To compare the test results of the novel timber construction systems with conventional timber construction methods, six tests with timber framed shear walls were carried out. Figure 10 shows the test setup for these walls. The walls have about the same height as PFTE and X-lam massive panel system walls. Due to the standard dimension of the sheathing (2.5 x 1.25 m), the length was set to 2.5 m so that two sheathing panels with one joint in the middle of the wall were used (Figure 10). Since PFTE wall elements are connected by staples on the chipboard side only, a "continuous" sheathing only exists on one side of the wall. The first test for conventional timber framed walls similarly was carried out with a wall sheathed on one side only. The results of the monotonic tests are shown in Figure 8.

Failure of the test specimen was reached by tensile failure of the OSB sheathing just above the hold-down. The first envelope curve of the cyclic test with the timber frame wall is nearly identical to the monotonic one (Figure 11), achieving about the same loads and displacements. The equivalent hysteretic damping v_{ed} for the first cycles is similar to the values achieved with PFTE, v_{ed} for the second and third cycle is significantly lower than the corresponding values of PFTE system.

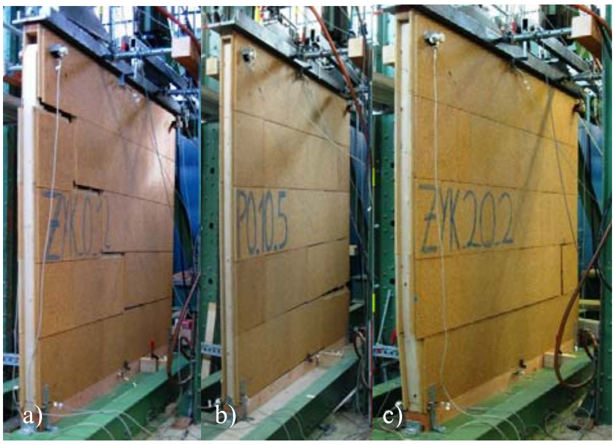

Figure 9: *Typical Failures in PFTE Tests*

The next four tests were conducted with the same test setup as shown in Figure 10, but with the wall being sheathed on both sides. Two tests were carried out with a vertical load of 1 kN/m and four tests with a vertical load of 10 kN/m, both with monotonic and cyclic loading.

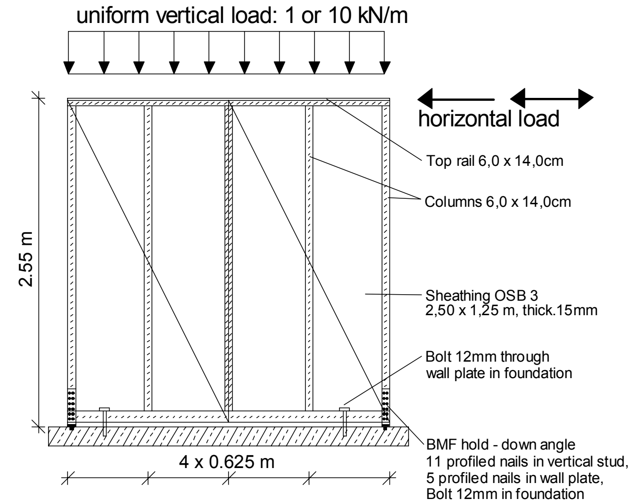

Figure 10: *Timber Framed Wall Test Specimen*

Table 1: *Comparison of Hysteresis equivalent damping*

	Hysteresis equivalent viscous damping 1st cycle	Hysteresis equivalent viscous damping 2nd and 3rd cycle
X-lam, staples	9.6% - 13.4%	8.4% - 10.9%
X-lam, nails	9.6% - 12.7%	9.3% - 11.9%
PFTE	13.9% - 15.7%	14.1% - 14.8%
Timber frame	10.9% - 12.9%	7.9% - 9.2%
Hysteresis equivalent damping $\nu_{ed} = \frac{E_d}{2\pi \cdot E_p}$		
where E_d = Dissipated Energy, E_p = Potential Energy		

As a result of the tensile failure of the OSB sheathing right above the hold-down, the hold-down was elongated for the final tests without vertical load. The total number of nails driven in the vertical stud was thus doubled. As can be seen in the results for both the monotonic and cyclic test, the performance of the wall was consequently improved.

4 NUMERICAL SIMULATION AND CALCULATION

The design of timber structures for earthquake actions according to Eurocode 8 [7] uses 3 ductility classes. Based on the ductile behaviour and the energy dissipation of the structure a behaviour factor q is defined. Using q, the actions on structures are reduced and lower forces have to be transmitted. This leads to a more realistic and cost-effective design.

4.1 BEHAVIOUR FACTOR Q

Timber constructions subjected to earthquake actions provide a number of advantages:

Related to its strength, timber has a low mass. During earthquake actions the mass excited to oscillations ("seismic mass") hence is lower than with other materials, resulting forces are thus smaller.

Mechanical fasteners behave in a ductile manner when being loaded in shear. This means that their failure mode is tough and plastic and not brittle. During earthquakes, this behaviour leads to structures suffering damages but not collapsing. The input kinetic energy is transformed by plastic deformation processes ("energy dissipation"). The energy is dissipated during (repeated) loading in the connections with mechanical fasteners.

This capability to resist actions above the elastic range should be considered when designing timber structures.

The behaviour factor q is approximately the ratio of forces loading the structure if its behaviour was completely elastic, to the forces on the structure incorporating plastic effects. Defining an ultimate displacement at which e.g. the withdrawal or the failure of single fasteners can be observed, a maximum load can be defined. At maximum load, failure of the connection is achieved. The behaviour factor q for earthquake loads therefore can be seen as:

$$q = \frac{R_{el}}{R_{pl}} \tag{1}$$

Where q = behaviour factor, R_{el} = Earthquake resistance assuming linear elastic behaviour, R_{pl} = Earthquake resistance considering plastic behaviour.

If a structure is designed so that it remains in the linear elastic level under earthquake loading and plastic behaviour is not taken into account, it should be assigned to ductility class "DCL" according to Eurocode 8 [7]. Structures in this ductility class e.g. are structures without or with only a few joints with mechanical fasteners, like cantilevers, beams, arches with two or three pinned joints or trusses joined with connectors. For these structures behaviour factor q should be taken to q = 1.5.

Structures can resist stronger earthquakes if the capability of plastic deformations is taken into account. In the design concept "dissipative structural behaviour", "…the capability of parts of the structure (dissipative zones) to resist earthquake actions above their elastic range is taken into account". Then "… the behaviour factor q may be taken as being greater than 1.5" (Eurocode 8 [7]). In the design concept "Medium capacity to dissipate energy" (Ductility class DCM) e.g. glued wall panels with glued diaphragms, connected with nails and bolts; Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill should be assigned. Then a behaviour factor q = 2 should be used. For hyperstatic portal frames with dowelled and bolted joints q = 2.5 should be used.

Using design concept "High capacity to dissipate energy" (Ductility class DCH), a behaviour factor q = 3 should be used for nailed wall panels with glued diaphragms connected with nails and bolts and for trusses with nailed joints.

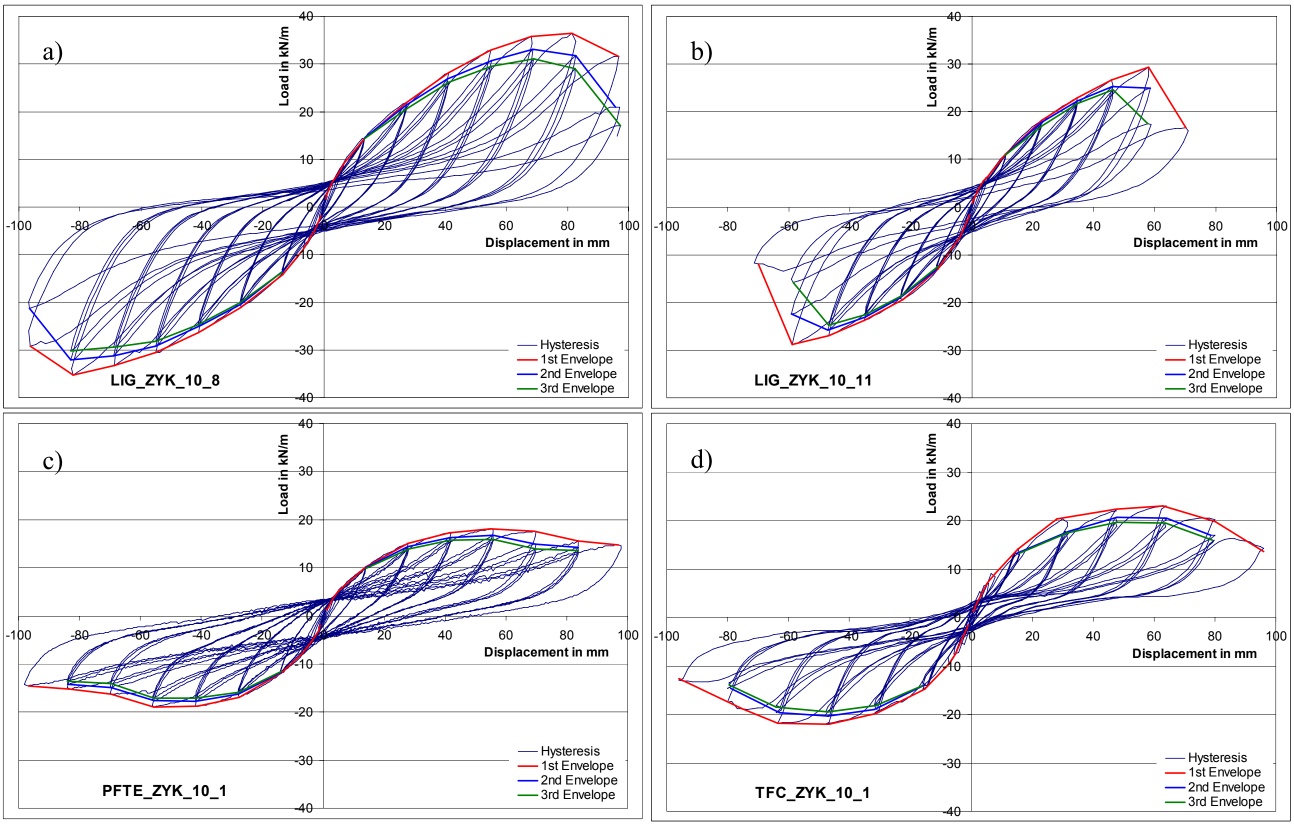

Figure 11: *Hysteresis for a) X-lam massive panel system connected with staples, b) X-lam massive panel system connected with nails, c) prefabricated timber wall elements, d) timber frame construction*

In the same design concept and ductility class, hyperstatic portal frames with doweled and bolted joints are classified but a behaviour factor q = 4 should be assigned. Using the same design concept and ductility class a behaviour factor q = 5 should be assigned to nailed wall panels with nailed diaphragms connected with nails and bolts.

4.2 NUMERICAL EVALUATION OF BEHAVIOUR FACTOR Q

The evaluation of the behaviour factor q for all systems is carried out with a model of a sample house reduced to a two-dimensional frame as shown in Figure 12.
This procedure (slightly modified) is taken from [8]. An XLAM building according to [8] was tested in shake table tests in June and July 2006. In [8], the behaviour factors q for the XLAM building were calculated numerically with a model calibration solely based on the hysteresis shape gained from cyclic testing. Comparing the numerical data with the results of the shake table tests, the excellent quality of the model can be seen.
The geometry in the presented paper was chosen as to compare test and numerical results. The essential properties of the system (ductility and energy dissipation) are taken into account in the simulation. To calculate the behaviour factor q, the model will be designed for a certain ground acceleration using force based design methods according to Eurocode 8 [7]. Thereafter the structure will be excited in each case by ten natural as well as ten artificial earthquakes (Table 2) and the response of the system will be calculated.

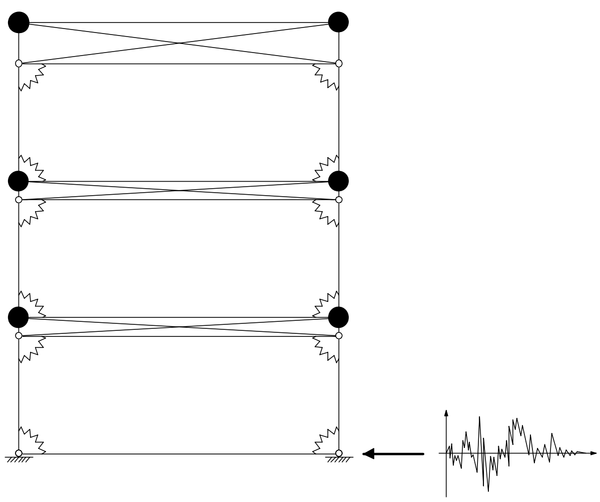

Figure 12: *Model chosen for the evaluation of behaviour factors*

The ratio of design ground acceleration to scaled ground acceleration in the model equals the behaviour factor q (Table 3). The artificial earthquakes were generated using the program SYNTH [9].

4.2.1 Course of action – short description

The approach to calculate the behaviour factors q is detailed in the following:

• According to Eurocode 8 [7] the earthquake actions for the structure are calculated for a ground acceleration of $a_g = 0.35g$ (equals 3.5 m/s^2). This acceleration is the peak value that has to be assumed for a building site in southern Europe (Italy) and in the following will be denoted „Peak Ground Acceleration“ ($PGA_{u,code}$).
• The distribution of horizontal seismic forces is calculated according to Eurocode 8 [7] while the behaviour factor is taken to be q = 1. Significantly increasing horizontal seismic forces follow from the assumption q = 1 compared to assuming q = 3 or q = 4.
• The wall lengths and so the required stiffness for the model are designed with the forces resulting from the assumption q = 1. The wall length needed for bracing the structure may be assumed as virtual length which is solely needed for linear-elastic calculation of the structure and for calibration of the model.

***Table 2:** Earthquakes for numerical simulation*

Location / Identifier of Earthquake	Date	Station	Component	Source	Duration in s
Roermond	13.04.1992	Bergheim	N/S	[9]	45
L'Aquila FA030x	06.04.2009	FA030	E/W	[13]	30
L'Aquila FA030y	06.04.2009	FA030	N/S	[13]	30
L'Aquila GX066x	06.04.2009	GX066	E/W	[13]	30
L'Aquila GX066y	06.04.2009	GX066	N/S	[13]	30
L'Aquila AM043x	06.04.2009	AM043	E/W	[13]	30
L'Aquila AM043y	06.04.2009	AM043	N/S	[13]	30
Friaul	06.05.1976	Feltre	N/S	[14]	33
Friaul	06.05.1976	Feltre	E/W	[14]	23
Lazio	07.05.1984	Atina	N/S	[14]	23
artificial quakes fitting the response spectra according to Eurocode 8 (a_g=3.5 m/s^2)					
SYNTH_1					15
…					15
SYNTH_6					15
…					15
SYNTH_10					15

• Based on the tests, the mechanical behaviour of the load-bearing walls is modelled using the computer program DRAIN-2DX [10]. The measured hysteresis loops are reproduced as closely as possible to capture the stiffness and dissipative properties of the walls. The walls are represented through a beam and column system and four non-linear springs placed in the corners. The hysteretic behaviour of the nonlinear springs is specified by the so-called *University of Florence model* [11].
• The model is loaded with real and artificial earthquake accelerograms. The accelerograms are scaled, so that at a specific scaling-value ($PGA_{u,eff}$) a default interstorey drift is reached.
• The division of the ground acceleration reached in the simulation and the calculated one represents the behaviour factor q.

$$q = \frac{PGA_{u,eff}}{PGA_{u,code}} \quad (2)$$

where $PGA_{u,eff}$ is the maximum ground acceleration at „near-collapse“ status and $PGA_{u,code}$ is the maximum ground acceleration given in the correspondent code.
"Near-collapse" here is an indicator for the structure being barely stable but suffering severe damage which e.g. can be a predefined interstorey drift.

The behaviour factor represents the structure's properties to withstand an earthquake which is several times stronger than in the chosen linear-elastic case. The q values reach different values for different earthquakes. A conservative value for q (e.g. the 5 % fractile value calculated in the simulation) represents the final behaviour factor q for the structure.

4.2.2 Determination of Near Collapse Status

The maximum amount of interstorey drift is taken as the abort criterion for the calculation.
The fundamental requirement for structures in seismic regions according to Eurocode 8 [7] is the no-collapse requirement. According to [7] "the structure shall be designed and constructed to withstand the design seismic action […] without local or global collapse, thus retaining its structural integrity and a residual load-carrying capacity after the seismic events. The design seismic action is expressed in terms of the reference seismic action associated with a reference probability of exceedance, PNCR, in 50 years or a reference return period, TNCR. The values recommended are PNCR = 10% and TNCR = 475 years. For the limitation of damage requirement, "…the structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action."
Both, PNCR and TNCR are comparable to the recommendations given in FEMA 450 [12]. Also the definitions of damage levels as well as the building categories described are comparable.
For building structures according to Seismic use group I, such as residential structures, FEMA 450 [12] allows a maximum interstorey drift of

$$u_{max} = 0.025\, h_{sx} = 0.025 \cdot 2570 \text{ mm} \cong 64 \text{ mm} \quad (3)$$

Where h_{sx} is the interstorey height (in (3) for the PFTE System).
An amount of 2.0 % of the storey height would be the maximum value for interstorey drift according to Eurocode 8 [7]. Due to the outstanding ductile behaviour of the tested systems the definition of the maximum interstorey drift being 2.5 % of storey height according to FEMA 450 [12] thus was taken into account.
An interstorey drift of 64 mm was obtained in the tests for all configurations while an amount of bearing capacity was still in the system.

5 OUTCOMES

5.1 Determination of behaviour factors

The stiffness and load-carrying capacity observed in the monotonic tests showed that the behaviour of both systems is similar to conventional timber construction systems. Also the hysteresis equivalent viscous damping ratio is in the same range. Therefore similar q-values for shear walls are expected when using these systems.
No q-values were determined for timber frame construction. Only one test with each configuration seemed to be insufficient to calculate a behaviour factor for a whole building system.

Table 3: *Calculated q-values for both systems*

Location / Identifier of Earthquake	$PGA_{u,code}$	PFTE $PGA_{u,eff}$	X-lam $PGA_{u,eff}$	PFTE q-value	X-lam q-value
natural earthquakes					
Roermond	0.35	1.59	1.34	4.5	3.8
L'Aquila FA030x	0.35	2.01	1.99	5.7	5.7
L'Aquila FA030y	0.35	1.89	1.56	5.4	4.5
L'Aquila GX066x	0.35	1.84	1.63	5.3	4.7
L'Aquila GX066y	0.35	1.54	1.54	4.4	4.4
L'Aquila AM043x	0.35	2.11	1.77	6.0	5.1
L'Aquila AM043y	0.35	1.75	1.72	5.0	4.9
Friaul	0.35	3.77	3.38	10.8	9.7
Friaul	0.35	3.68	3.30	10.5	9.4
Lazio	0.35	1.29	1.10	3.7	3.1
artificial earthquakes					
SYNTH_1	0.35	1.44	1.68	4.1	4.8
SYNTH_2	0.35	1.51	1.55	4.3	4.4
SYNTH_3	0.35	1.58	1.36	4.5	3.9
SYNTH_4	0.35	1.29	1.26	3.7	3.6
SYNTH_5	0.35	1.46	1.29	4.2	3.7
SYNTH_6	0.35	1.16	1.40	3.3	4.0
SYNTH_7	0.35	1.58	1.41	4.5	4.0
SYNTH_8	0.35	1.51	1.23	4.3	3.5
SYNTH_9	0.35	1.49	1.51	4.3	4.3
SYNTH_10	0.35	1.50	1.15	4.3	3.3
		average value		5.1	4.7
		5% fractile		3.7	3.3
		=> q-value		**4.0**	**3.0**

When subjected to cyclic loads the systems showed favourable characteristics and a large amount of energy dissipation. The q-values for the PFTE- System are only in three cases lower than q = 4, while the 5 %-fractile is 3.7 (Table 3). In the chosen configuration the maximum interstorey drift is always reached in the first storey. Regarding the first floor of a three-storey building, the assumption of an upload of 10 kN/m is conservative. First floor walls will generally have higher uploads. Because of the higher amount of energy dissipation when subjected to higher uploads a value of q = 4.0 is recommended for the PFTE-system. The q-value for the X-lam massive panel system in no case falls below a value of q = 3 while the 5%-fractile is 3.3. Therefore a value of q = 3.0 is recommended for the X-lam massive panel system.

5.2 Critical reflection of course of action

When calculating structures for seismic areas the European approach is “traditionally” force-based. This means that based on ground types, ground acceleration, seismic use groups and other determining factors, horizontal seismic forces are calculated that act on the structure. Generally the main attention is focused on the verification of the ultimate limit state; the verification of the damage limitation state usually is neglected.

By contrast to this, displacement-based design procedures have been developed in the past few years. These procedures (Performance Based Seismic Design (PBSD) [15], Direct Displacement Design (DDD) [16], and Capacity Spectrum Method [9]) are using a pre-defined target displacement to calculate the required stiffness of a building taking into account the damping of the structure as well as the hazard level This leads to structures suffering only minor damage in lighter earthquakes while the safe evacuation of the occupants in stronger earthquakes is ensured.

Due to not explicitly considering hazard levels and drift limits, force-based design methods are criticised in literature. The main critical matters of discussion when regarding force-based design methods according to [15] are the following:

- Since the determined q-values for the different materials and constructions are primarily based on judgement, they are difficult to justify when using force-based methods. Without knowledge of the global system response, q-factors cannot be rationally determined.
- Deformation limit states are not directly addressed by the force-based design procedure. Limiting deformations is paramount for wood framed buildings since a large portion of the damage to wood framed buildings is associated with excessive lateral displacements.
- q-Factors are associated with the global displacement capacity of the structure. The displacement ductility is based on the ratio of ultimate displacement to first-yield displacement. There is a major difficulty in reaching consensus on the appropriate definition of yield and ultimate displacements for wood framed lateral load-resisting systems.

Some arguments considered by the authors may challenge the chosen course of action using the numerical simulation within this paper additionally:

- Calibrating the hysteresis, the chosen model depends on engineering judgement as well. The stiffness and displacement chosen for the calibration both have a large influence on the response of the structure. The correlation of energy dissipation theoretically can be achieved using a completely different calibration.
- Solely the behaviour of the shear walls is considered by taking into account their maximum horizontal bearing load. The bracing of the walls as well as the floors are assumed to be rigid. No torsional effects are considered within this paper.

Opposite to this, the advantages of the chosen course of action that led to the usage in this research project are:

- The method is more precise than the declaration of a static ductility as given in Eurocode 8 because the energy dissipation of the substructure is taken into account. No displacement ductility is needed.
- Using the described simulation, basic values for the behaviour factor q can be estimated comparatively fast and easy. The q-values are verified through the conservative testing on which the model is calibrated.
- The chosen methodology can be broadened to a 3d-Model [8] if needed. Torsional effects and other details can be taken into account.
- Based on few and common tests on shear walls an essential statement regarding the behaviour of the system can be given. Complex testing can be omitted. Regarding innovative timber systems, design fundamentals can be given in a short time. This increases market opportunities of novel construction systems for the application in seismic prone areas throughout the world.
- The estimated q-values are not based on engineering judgement, but are calculated and verified using 20 accelerograms. Using this multiplicity of accelerograms, the calculated value has a broad basis.

6 CONCLUSIONS

The advancement and research of two innovative timber construction systems is presented in this paper. Both systems as well as the established timber construction system have been tested under monotonic and reversed-cyclic loading. Using a numerical simulation, the properties of the systems subjected to earthquake loading were reproduced and the behaviour factor q was calculated. Basic principles for the calculation and construction with the systems in seismic areas are thereby developed. Market opportunities for innovative construction systems can be seized quicker using these investigations carried out previously.

The massive X-lam panel system showed good performance for both monotonic and cyclic testing, the maximum horizontal load was even better than for the tested timber frame system. The energy dissipation for the X-lam massive panel system is about the same than for the timber frame system.

The system with PFTE showed good performance in monotonic and cyclic testing as well. In monotonic tests the results for maximum horizontal load and for stiffness values are quite similar to conventional timber frame systems. PFTE showed excellent results for the energy dissipation in cyclic loading. Further work is being done to improve the hold-down of the vertical tensile studs. The PFTE and the X-lam system can cover the same application range as conventional timber frame buildings, yet they are easy to handle and therefore cost effective.

Both systems are highly suitable for the use in seismic areas, while their behaviour can be classified similar to the well-known timber frame system. The maximum loads as well as the measured displacements equal or exceed the respective values of the conventional timber frame system.

Future research work will be devoted to finite–element models to simulate both system properties in more detail to reduce the number of tests needed.

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REFERENCES

[1] Dujic, B., Aicher, S., Zarnic, R., 2005. „Investigations on in-plane loaded wooden elements – influence of loading and boundary conditions" Otto-Graf-Journal Vol. 16, 2005, pp 259 – 272.

[2] EN 594, 1996. Timber Structures – Test methods – Racking strength and stiffness of timber frame wall panels.

[3] EN 12512, 2001. Timber Structures – Test methods – Cyclic testing of joints made with mechanical fasteners.

[4] ISO Standard 16670, 2003. Timber Structures – Joints made with mechanical fasteners – Quasi-static reversed-cyclic test method.

[5] Committee Draft ISO/CD 21581, 2007. Timber Structures – Static and cyclic lateral test method for shear walls

[6] Blaß, H. J.; Schädle, P. (2009): Aussteifende Wände in Einzelelement-Bauweise. Band 13 der Reihe Karlsruher Berichte zum Ingenieurholzbau. Universitätsverlag Karlsruhe, 2009

[7] Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings; German version EN 1998-1:2004

[8] Ceccotti, A. (2008): New Technologies for Construction of Medium-Rise Buil-dings in Seismic Regions: The XLAM Case. In: Structural Engineering International (IABSE), May 2008, S. 156 -165

[9] Meskouris, K.; Hinzen, K.-G.; Butenweg, C.; Mistler, M. (2007): Bauwerke und Erdbeben. Vieweg Verlag Wiesbaden, 2007

[10] Prakash et al. (1993): DRAIN-2DX Base Program Description and User Guide, Version 1.10, University of California, Berkeley, 1993

[11] Ceccotti, A.; Vignoli, A. (1989): A hysteretic behavioural model for semi-rigid joints. European Earthquake Engineering, 3, 1989

[12] National Institute of Building Sciences: National Earthquake Hazards Reduction Program – Recommended provisions for seismic regulations for new buildings and other structures (FEMA 450), 2003

[13] www.reluis.it Homepage of "The Laboratories University Network of seismic engineering (ReLuis)"

[14] http://peer.berkeley.edu/nga/earthquakes.html Homepage of Pacific Earthquake Engineering Research Center, NGA Database

[15] Filiatrault, A.; Folz, B. (2002): Performance-Based Seismic Design of Wood Framed Buildings. Journal of Structural Engineering, ASCE, Vol. 128, No. 1, January 2002

[16] Pang, W.; Rosowsky, D. (2007): Direct Displacement Procedure for Performance-Based Seismic Design of Multistory Woodframe Structures, Zachry Dept. of Civil Engineering, Texas A&M University, www.engr.colostate.edu/NEESWood